SHORT POURED-IN-PLACE CONCRETE PILES

TO RESIST UPLIFT AND LATERAL LOADS

INTRODUCTION

Occasionally it is necessary to determine the capacity of short poured-in-place concrete piles used to resist uplift and lateral loads. The following is a brief review of the technical aspects and a procedure which can be used for investigating rigid piles. This is not a comprehensive coverage of the subject - there are soil complexities not covered, and some caution should be used in its application if primary loads of extended duration are to be supported.

The pile must have the structural capacity to resist tensile, shear and bending stresses. Reinforcing steel should extend the full length of these piles.

The Office of Geotechnical Engineering of the Division of New Technology, Materials and Research in Sacramento has furnished the Office of Structure Construction criteria for the analysis of loadings on poured-in-place concrete piles. The analysis is dependent on proper selection of soil type. It will be important to determine whether the soil into which the pile is constructed is principally cohesive or cohesionless. Analytical results for pile uplift and lateral loading represent ultimate resistance values.

Load resisting capacity is dependent upon the characteristics of the soil into which the pile has been cast. Preliminary assumptions may be made about soil properties at the time of review of the working drawings. A final determination of the pile's capacity should be made, however, when the pile hole is excavated and the actual soil can be inspected. The type of soil in the upper third of the hole, its degree of compaction and whether ground water is (or may be), encountered are of primary importance.

Pile loadings are considered in three separate categories; Pile Uplift, Lateral Loads, and Resistance to Combined Uplift and Lateral Loads - all with sample problems.

PILE UPLIFT

Pile uplift, acting either vertically or at an angle is resisted by soil-pile friction (shearing resistance) and the physical weight of the pile.

The shearing resistance of the soil-pile interface is computed differently for cohesive soils than it is for cohesionless soils. The internal angle of friction of the soil is not utilized for poured-in-place piles because in hard ground (high friction angle) the drilling operation loosens the adjacent soil, and in loose ground (low friction angle) the drilling operation tends to compact the adjacent soil particles.

Ultimate pile resistance to uplift is determined by adding the weight of the pile to the quantity of the appropriate unit shearing resistance value multiplied by the surface area of the pile. No additional provisions are made for irregularities along the pile-soil interface.

Resistance to pile uplift = πdzS + pile weight

where:
- d = Pile diameter
- z = Depth below ground surface
- S = Unit shearing resistance on the soil-pile interface, psf

Generally, working load values are to be limited to no more than one-half the ultimate load values, which should provide a minimum safety factor of 2.

Pages C-9-2 through C-9-3 illustrate pile uplift in cohesionless type soils and pages C-9-4 through C-9-5 illustrate pile uplift in cohesive soils.

PILE UPLIFT IN-COHESIONLESS SOIL

For cohesionless soil, the soil-pile friction (shearing resistance) may be computed using the following equation:

$$S = \beta\sigma_z \text{ psf} \quad \text{but } S \leq 4{,}000 \text{ psf}$$

where:
- **β = $1.5 - 0.315(z)^{1/2}$ but $0.25 \leq \beta \leq 1.2$. A unitless reduction factor for cohesionless soils.**
- **σ_z = The effective overburden soil weight. Below the water table the weight of water is subtracted from the soil unit weight so that only the submerged soil weight is used.**

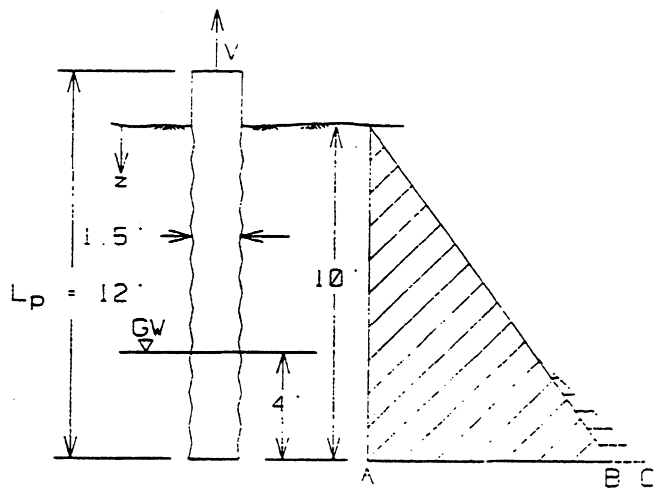
EXAMPLE PROBLEM, PILE UPLIFT IN COHESIONLESS SOIL

Determine the vertical load capacity of an 18 inch diameter poured-in-place concrete pile 12 feet long embedded vertically 10 feet in the ground. The water table will rise about 4 feet up from the pile tip at the anticipated time of load application. Soil parameters are as follows:

Soil internal friction angle ϕ = 30°

Unit weight of concrete
γ_c = **145 pcf**

Unit weight of soil
γ_s = **100 pcf**



Where:

AB = The pressure due to the weight of the soil.
BC = The pressure due to the weight of the water.
L_p = Length of the pile.
d = Pile diameter = 1.5 feet.

Solution:

Unit shearing resistance:

$$\beta = 1.5 - 0.315(z)^{1/2} = 1.5 - 0.315(10)^{1/2} = 0.5 \quad \text{O.K.}$$

$$z = 10' \qquad \text{but } z_{dry} = 6' \text{ and } z_{submerged} = 4'$$

$$\sigma_z = 6(100) + 4(100 - 62.4) \approx 750 \text{ pcf}$$

$$S = \beta\sigma_z = 0.5(750) = 375 \text{ psf} < 4{,}000 \text{ psf} \quad \text{O.K.}$$

Net pile shearing resistance = (Pile surface area)S
= $\pi dzS = \pi(1.5)(10)(375)$

= 17,671 Lbs

$$\text{Pile weight} = \frac{\pi d^2 L_p \gamma_c}{4} = \frac{\pi(1.5)^2(12)(145)}{4} = 3{,}075 \text{ Lbs}$$

Ultimate load capacity = Net pile shearing resistance + Pile weight
= 17,671 + 3,075 = 20,746 Lbs

$$\text{Working load (V)} = \frac{\text{Ultimate load}}{2} = \frac{(20{,}746)}{2} = 10{,}373 \text{ Lbs}$$

PILE UPLIFT IN COHESIVE SOIL

The soil-pile friction equations for cohesive soils differ substantially for pile penetrations of less than 5 feet versus piles over 5 feet in depth. The equations also depend on whether the pile is greater or less than 18 inches in diameter.

Shearing resistance = πdzS

Where $S = \alpha_z C$

S = Unit shearing resistance, but $S \leq 5{,}500$ psf.
C = Soil cohesion (undrained shear strength).
z = Depth below ground surface in feet.
d = Diameter of the pile.
α_z = An empirical unitless reduction factor derived from load testing which accounts for clay shrinkage and lateral pile loadings. This variable depends on the depth of pile penetration, having one value for a depth up to 5 feet, and another for penetration over 5 feet.

Reduction factor α_z for pile diameters (d) greater than 18":

The reduction factor α_z for the first 5 feet of penetration is 0. The reduction factor remains constant at α_z = 0.55 for all depths greater that 5 feet. This may be expressed in equation form as:

1. For short piles, 5' or less embedment:

$$\alpha_{z(0-5)} = 0 \qquad \text{for} \quad 0 \leq z \leq 5 \text{ feet}$$

2 . For pile lengths with 5' or more embedment:

$$\alpha_{z(0-5)} = 0 \qquad \text{for} \quad 0 \leq z \leq 5 \text{ feet}$$

$$\alpha_{z(>5)} = 0.55 \qquad \text{for} \quad z > 5 \text{ feet}$$

Reduction factor α_z for pile diameters (d) 18" or less:

The reduction for the top 5' of pile varies from 0 at z = 0' to 0.55 at z = 5', then remains constant at 0.55 for all depths greater than 5'. For lengths of pile between 0 and 5', prorate the reduction factor. This concept may be expressed in equation form as:

1 . For short piles, 5' or less embedment:

$$\alpha_{z(0-5)} = \left(\frac{0 + 0.55}{2}\right)\frac{z}{5} = (0.275)\frac{z}{5}$$

$$= (0.055)z$$

2. For pile lengths with 5' or more embedment:

$$\alpha_{z(0-5)} = \left(\frac{0 + 0.55}{2}\right)\frac{z}{5} = (0.275)\frac{z}{5}$$

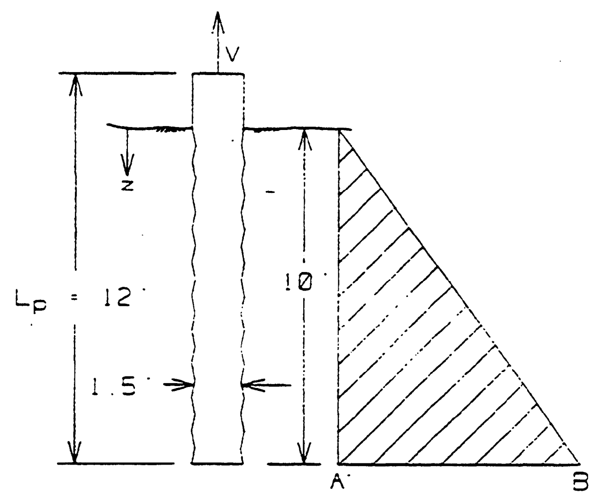
$$= (0.055)z$$

$$\alpha_{z(>5)} = 0.55$$

EXAMPLE PROBLEM. PILE UPLIFT IN COHESIVE SOIL

Determine the vertical load capacity of an 18 inch diameter poured-in-place concrete pile 12 feet long embedded vertically 10 feet in-cohesive soil with an average undrained shear strength value of 910 psf where the concrete unit weight γ_c is 145 pcf and the soil unit weight γ_s is 110 pcf.

Solution:



$$\alpha_{z(0-5)} = (0.055)z$$

$$= (0.055)5 = 0.275$$

$$\alpha_{z(>5)} = 0.55$$

$$S_{(0-5)} = \alpha_{z(0-5)}C \text{ psf}$$

$$= 0.275(910) = 250 \text{ psf}$$

$$S_{(>5)} = \alpha_{z(>5)}C$$

$$= 0.55(910) = 500 \text{ psf}$$

Where:
- AB = The pressure due to the weight of the soil.
- L_p = Length of the pile.
- d = pile diameter = 1.5 feet.

Net shearing resistance $= \pi d[(5)S_{(0-5)} + (z-5)S]$

$$= \pi(1.5)[(5)(250) + (10-5)(500)]$$

$$= \pi(1.5)(3750) = 17{,}671 \text{ Lbs}$$

Pile weight $= \frac{\pi d^2}{4}L_p\gamma_c = \frac{\pi(1.5)^2}{4}(12)(145) = 3{,}075$ **Lbs**

Ultimate load capacity = Net pile shearing resistance + Pile weight
= 17,671 + 3,075 = 20,746 Lbs

Working load (V) $= \frac{\text{Ultimate load}}{2} = \frac{20{,}746}{2} = 10{,}373$ **Lbs**

LATERAL LOADS

Tests have shown that soil resistance to lateral pile loading is greater than that predicted by Rankine equations. For clays the ultimate passive resistance can be as large as 9 times the shear strength (C), and for cohesionless soils the ultimate resistance can be 3 times as large as computed Rankine values. The soil resistance acting on isolated piles to a lateral force applied at or near the ground surface may be somewhat depicted as shown in Figure 1.

Convenient concepts and equations have been developed by Broms for cohesive and cohesionless soils. If a few important soil properties are known, or Can be determined, it is possible to compute soil resisting values and pile moments resulting from the application of lateral pile forces at or near the ground surf ace.

Ultimate pile resistance to lateral loading may be determined by failure of the soil along the total pile length in the case of short piles, or by the yield moment of the pile itself for longer piles. Short unrestrained piles are those piles having a length to diameter ratio (L/d) of 20 or less providing the yield moment (M_y) of the pile will be greater than the maximum resisting moment (M_{ULT}) furnished by the soil.

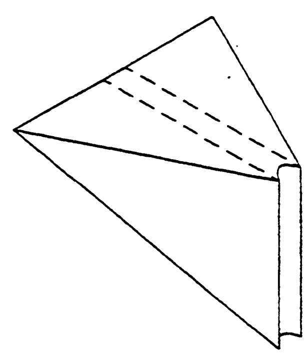


FIGURE 1

Embedment of piles should be a minimum of 4 times the pile diameter to achieve sufficient soil resisting capacity. The point of rotation of rigid short piles may be assumed to occur between 0.70 to 0.75 times the embedded length; where the larger value coincides with the largest lateral loadings. Soil resisting values are determined by using the lateral resisting value of up to 3 times the passive coefficient K_P for cohesionless soil and as much as 9 times the undrained shear strength C_u of cohesive soils.

Piles may be considered to act individually provided the pile spacing exceeds 4 pile diameters. When piles are spaced closer than 2 pile diameters, the piles and the soil within the pile group may be considered to act as a single unit.

As piles under load deflect they place the forward soil in a passive condition. When a pile is in clay, a void will be left behind the pulled pile until the clay crumbles or swells. When a pile is in granular material, the soil will soon fill the void behind the pulled pile. When a pile is unloaded, it will generally not return to its original position; some of the pulled deflection will remain. It can readily be seen that unloading and reloading a pile greatly decreases the soils moment capacity for that pile.

Generally, working load values are to be limited to no more than one-half the ultimate load values, which should provide a minimum safety factor of 2 assuming a one time loading of the soil around the pile.

For each subsequent time a pile is to be loaded in the same direction, an additional safety factor of 0.25 is to be added to the previous value as defined by the following:

$$SF = 2 + (X-1)(0.25)$$

where:

SF = Safety factor

x = Number of uses in the same direction for the same horizontal component.

LATERAL LOADING IN COHESIONLESS SOILS

Considerations used for piles in-cohesionless soil include increasing the Rankine passive resistance by a factor of 3, ignoring active pressures on the back side of the pile, and assuming that soil along the total length of buried pile provides resistance at the moment of loading.

Figure 2 depicts soil pressure diagrams for short and long isolated piles in cohesionless soil. The passive resistance at the toe of the short piles is replaced by a concentrated load acting at the pile tip to simplify the moment equation. A plastic hinge is assumed for long piles and the maximum bending moment will be limited to the yield moment (M_Y) of the pile.

The maximum moment for short piles occurs at the location of zero shear. For granularsoils this plane of zero shear is located at a pile depth of $e + f_g$ below the plane of application of the lateral load. The distance f_g equals the length from the ground surface to the plane of zero-shear.

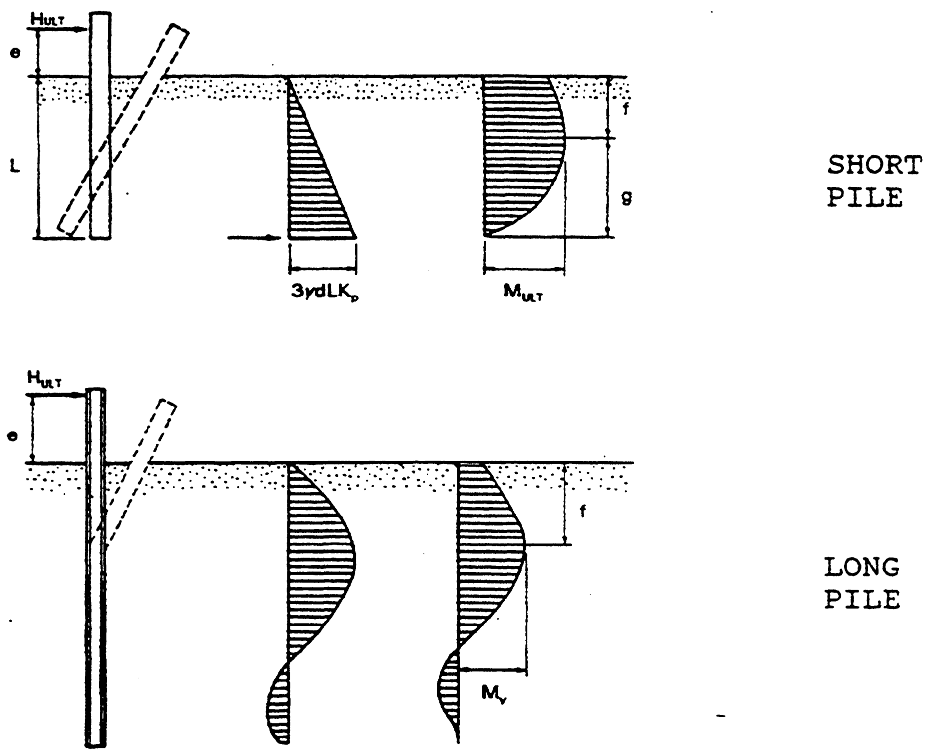


FIGURE 2

Equating lateral forces gives:

$$(f_g)^2 = \frac{H_{ULT}}{1.5\gamma_S dK_P}$$

The maximum moment occurs at a depth of e + f_g:

$$M_{ULT} = H_{ULT}\left(e + \frac{2f_g}{3}\right)$$

Based on failure of the soil.

If the moment M_{ULT} is calculated to be greater than the pile limiting yield moment M_Y a long pile is indicated and therefore H_{ULT} must be limited by using $M_{ULT} = M_Y$.

Figure 3 contains curves developed by Broms which relate the pile embedment length ratio L/d to the ultimate lateral soil resistance for various e/L ratios. H_{ULT} can be determined for short piles by using Figure 3.

Figure 4 may be used for long piles. Broms' curves for values of e/d relate the soil ultimate lateral resistance to the yield moment of the pile. This figure is used when the pile embedment length ratio L/d is greater than 20 and when the

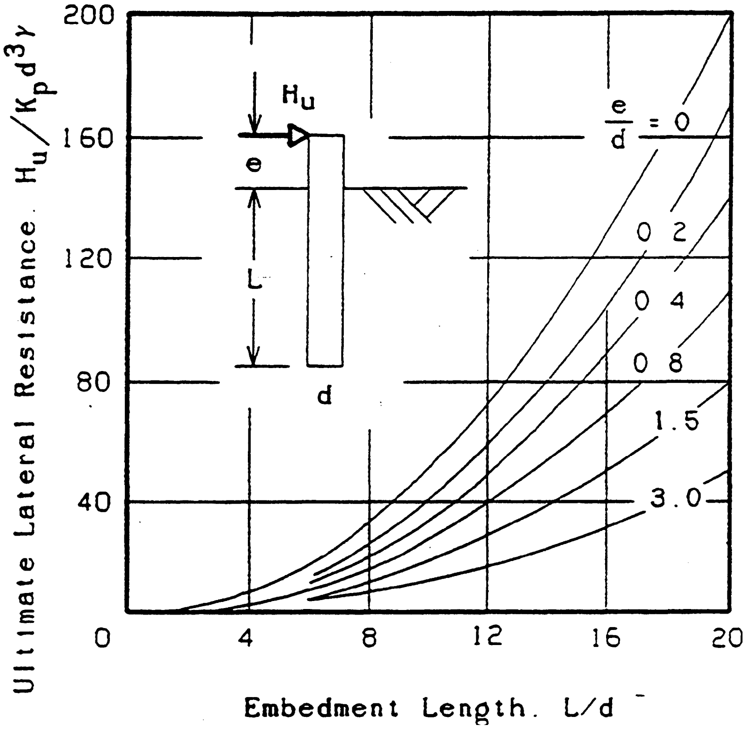


FIGURE 3

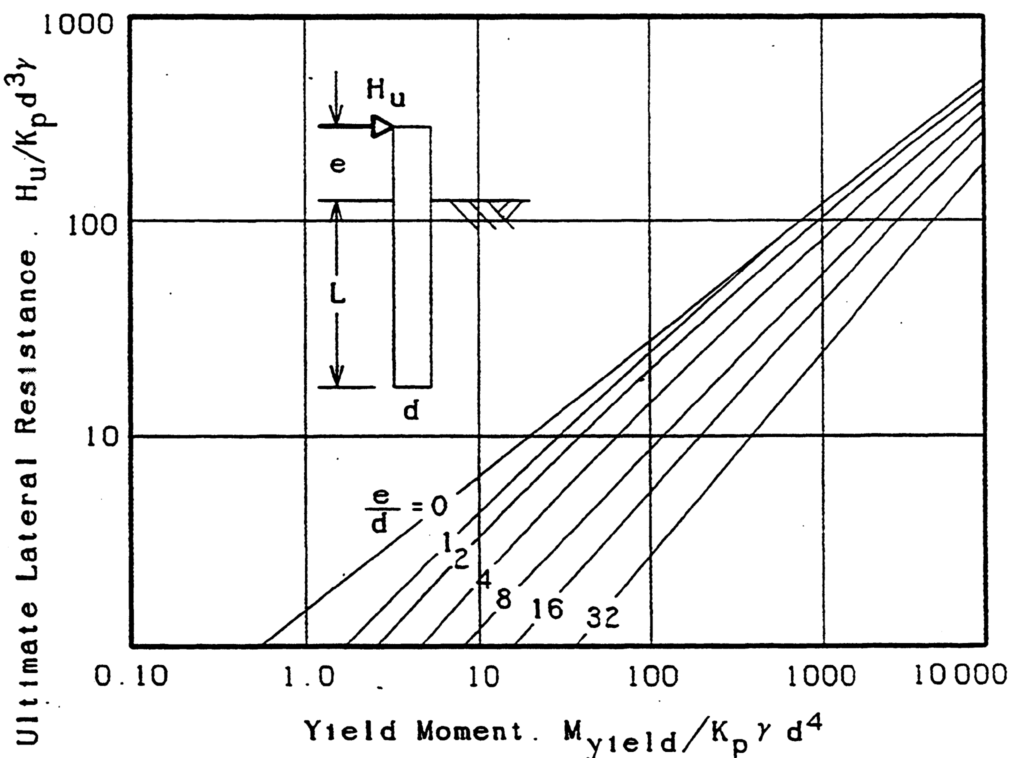
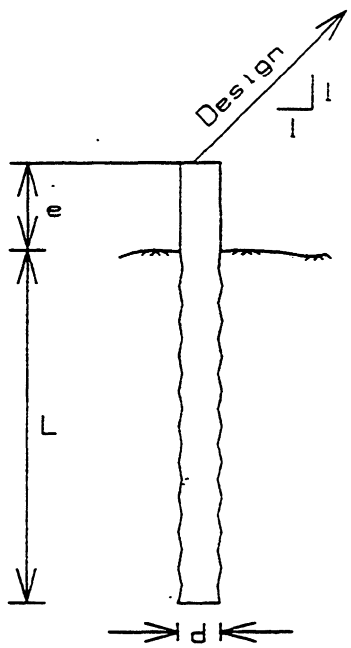


FIGURE 4

yield moment of the pile is less than the ultimate lateral soil resistance.

The maximum safe single use working load for free headed piles in cohesionless soils may be taken as one-half of the ultimate load values.

EXAMPLE PROBLEM, LATERAL LOADING IN COHESIONLESS SOIL



Design = 3,800 Lbs
H = 2,687 Lbs
V = 2,687 Lbs

$\phi = 30°$
$\gamma_s = 110$ pcf
$\gamma_c = 145$ pcf
L = 8'-0"
pile d = 1'-6"
e = 2'-0"

Single use loading

Solution:

$K_p = \tan^2(45° + \frac{\phi}{2})$ = 3.00 (for level ground surface)

L/d = 5.33 e/d = 1.33

From Figure 3:

$$\frac{H_{ULT}}{K_p \gamma d^3} \approx 5 \quad \text{when } e = 2'\text{-}0"$$

$H_{ULT} = K_p \gamma d^3 (5) = (3.0)(110)(1.5)^3(5) = 5{,}569$ Lbs

Working Load Value for H = $\frac{5{,}569}{2}$ = 2,784 > 2,687 Lbs

Compute f_g and M_{ULT}:

$$f_g = \left[\frac{H_{ULT}}{1.5\gamma_s d K_p}\right]^{1/2} = \left[\frac{5{,}569}{1.5(110)(1.5)(3.0)}\right]^{1/2} = 2.74 \text{ feet}$$

$$M_{ULT} = H_{ULT}\left[e + \frac{2f_g}{3}\right] = 5{,}569\left[2 + \frac{(2)(2.74)}{3}\right] = 21{,}311 \text{ Ft-Lb}$$

Working Load Value for $M = \dfrac{21{,}311}{2} =$ 10,656 Ft-Lbs

LATERAL LOADING IN COHESIVE SOILS

The ultimate soil resistance for piles in cohesive soils increases to some maximum value at approximately 3 pile diameters below the ground surface then remains fairly constant at greater depth. Literature suggests using a soil distribution of zero between ground surface and a depth of 1.5 times the pile diameter (1.5d) and then using a value of 9 times the undrained shear strength $(9C_u)$ for the remainder of the pile depth.

SHORT PILE

LONG PILE

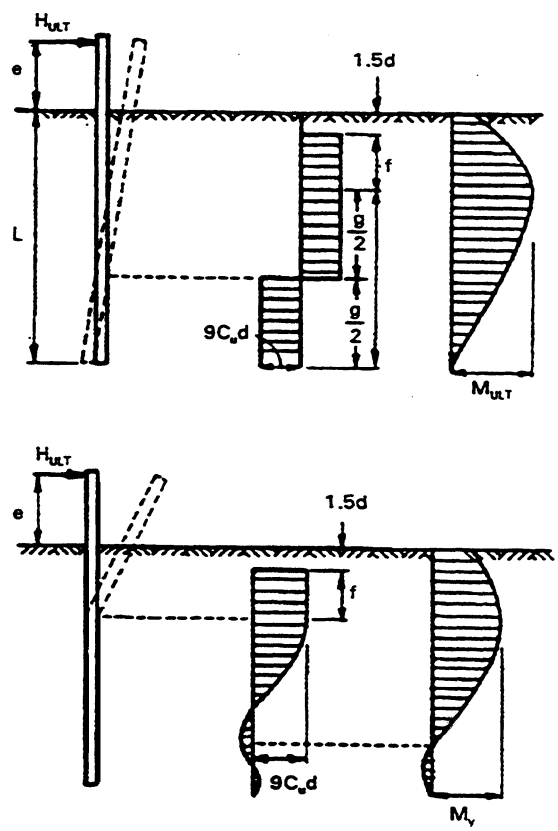


FIGURE 5

Figure 5 depicts soil pressure diagrams for short and for long piles in cohesive soils. Short piles have a limiting embedment length ratio of L/d = 20. Piles having L/d ratios in excess of 20 are considered to be long piles. For long piles a plastic hinge is assumed in the vicinity of the maximum moment. The yield moment M_Y of long piles will generally limit the soil resisting maximum moment M_{ULT} so that $M_{ULT} = M_Y$ should be used.

The maximum moment for short piles occurs at the location of zero shear. For cohesive soils the plane of zero shear is located at a pile depth of e + 1.5d + f_c below the plane of application of the horizontal force. The distance f_c develops from equating horizontal forces:

$$f_c = H_{ULT}/9C_ud$$

The maximum moment occurs at a depth of e + 1.5d + f_c:

$$M_{ULT} = H_{ULT}(e + 1.5d + 0.5f_c)$$

Based on failure of the soil.

If the moment M_{ULT} is calculated, to be greater than the yield moment M_Y of the pile, a long pile is indicated and H_{ULT} must be limited by using $M_{ULT} = M_Y$.

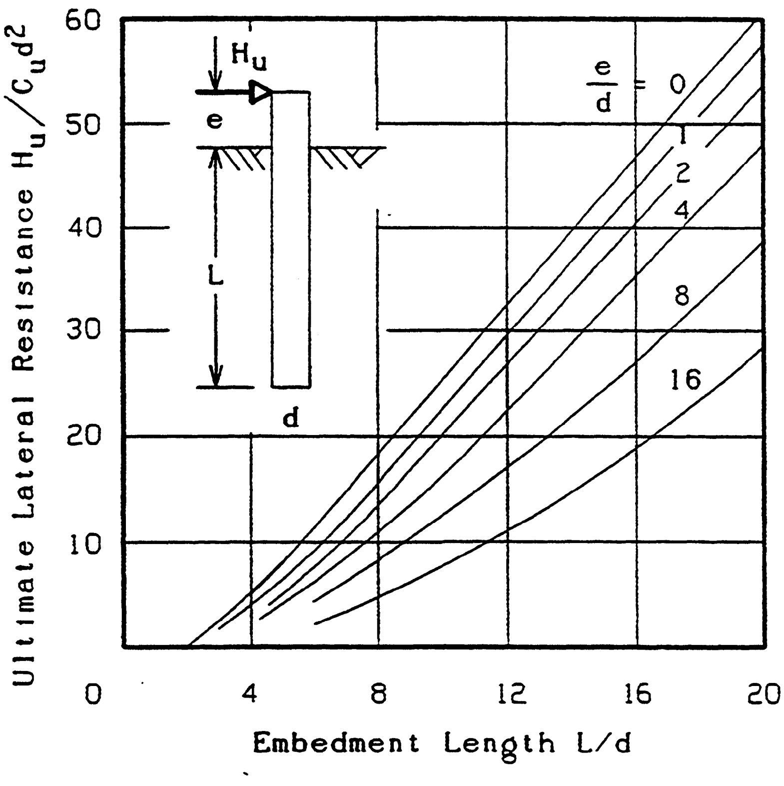


FIGURE 6

Figure 6 contains curves developed by Broms for short piles which relates the pile embedment depth ratio L/d to the ultimate lateral soil resistance for various e/d ratios.

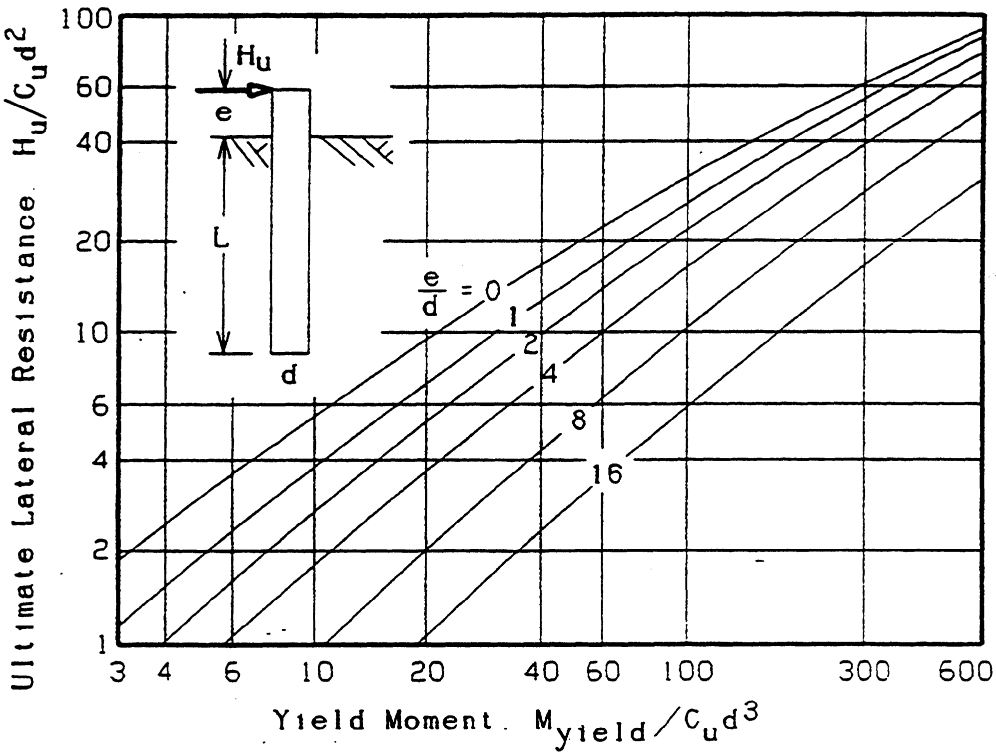
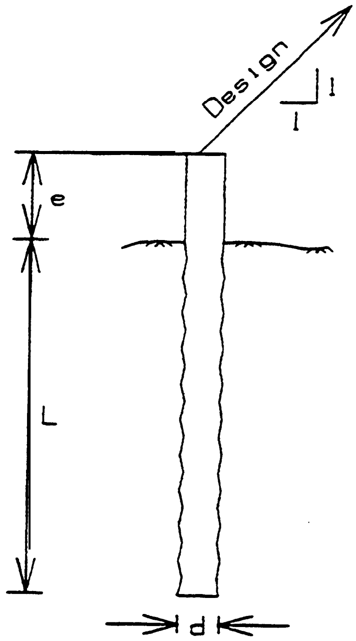


FIGURE 7

Figure 7 may be used for long piles. Curves developed by Broms for e/d values relate the soils ultimate lateral resistance to the yield moment of the pile. This figure is used when the pile embedment length ratio L/d is greater than 20 and when the yield moment of the pile is less than the moment due to the ultimate lateral soil resistance.

The safe single use working load for free headed piles in cohesive soil may be taken as one-half of the ultimate load value.

EXAMPLE PROBLEM, LATERAL LOADING IN COHESIVE SOIL



C_u = 1,000 psf
γ_s = 110 pcf
γ_c = 145 pcf
L = 8′-0"
pile d = 1′-6"
e = 2′-0"

Single use loading

Determine the allowable loading for this pile:

Solution:

L/d = 5.33 e/a = 2/1.5 = 1.33

From Figure 6:

$$\frac{H_{ULT}}{C_u d^2} \approx 5.5 \text{ when } e = 2' - 0"$$

$$H_{ULT} = C_u d^2(5.5) = (1{,}000)(1.5)^2(5.5) = 12{,}375 \text{ Lbs}$$

Working Load Value for H $= \frac{12{,}375}{2} =$ 6,188 Lbs

Compute f_c and M_{ULT}:

$$f_c = \frac{H_{ULT}}{9C_u d} = \frac{12{,}375}{9(1{,}000)(1.5)} = 0.917 \text{ feet}$$

$$M_{ULT} = H_{ULT}\left[e + 1.5d + \frac{f_c}{2}\right] = (12{,}375)[\ 2 + 2.25 + 0.46\ ]$$

$$= 58{,}266 \text{ Ft-Lb}$$

Working Load Value for M $= \frac{58{,}266}{2} =$ 29,133 Ft-Lbs

CONCRETE STRESSES

Concrete stresses in the pile may be computed by rigorous analysis; or may be approximated by assuming an average compressive condition over one-half of the pile width. The maximum compressive stress is located on one face of the pile. It should be assumed that the concrete will not take tensile forces on the other half of the pile. Tensile forces will be resisted by reinforcing steel.

For simplified compressive analysis use:

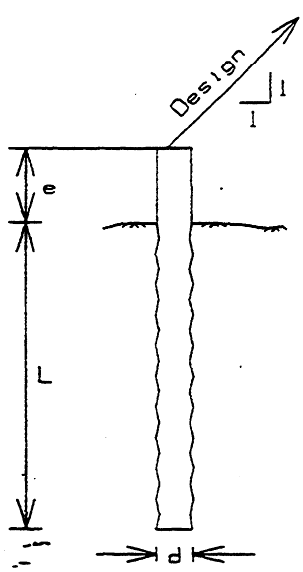
$$\textbf{Compressive stress } f_c = \frac{Md}{I_g 2} - \frac{V'}{A_g}$$

Where: d = Pile diameter.
I_g = Moment of Inertia on the gross pile section.
A_g = Gross cross-sectional pile area.
V' = Tensile force (vertical force component) less pile weight above plane of zero Shear. Distance from pile top to the plane of zero shear is defined as M_{ULT}/H_{ULT}.

The computed maximum compressive stress f_c shall not be greater than one-half of the concrete cylinder strength (f'_c) anticipated at the time the pile is to be loaded.

The allowable shear in the pile (V_U) normal to the pile should not exceed 2 times the square root of f'_c $(2\sqrt{f'_c})$.

EXAMPLE PROBLEM, CONCRETE STRESS



V_{MAX} = 6,188 Lbs
H_{MAX} = 6,188 Lbs
M_{MAX} = 29,133 Ft-Lbs
L = 8'-0"
pile d = 1'-6"
e = 2'- 0"
f'_c = 3,250

Single use loading

Determine the concrete stress for this pile:

Solution:

With forces acting through the center of the pile consider one-half of pile in compression.

Use the simplified equation:

$$f_c = \frac{Md}{2I_g} - \frac{V'}{A_g}$$

where V' = 6,188 minus the pile weight above the plane of zero shear.

$$\text{Distance to plane of zero shear} \approx \frac{M_{ULT}}{H_{ULT}} \approx \frac{M_{MAX}}{H_{MAX}}$$

$$\approx \frac{29,133}{6,188} \approx 4.7 \text{ feet}$$

$$\text{Pile Weight} = (4.7 + 2)(\gamma_c)(\pi)(\frac{d}{2})^2$$

$$= 6.7(145)(\pi)(\frac{1.5}{2})^2 = 1,717 \text{ Lbs}$$

V' = 6,188 - 1,717 = 4,471 Lbs

$$f_c = \frac{29,133(12)(1.5)(12)}{2(5153.0)} - \frac{4,471}{254.5}$$

$$= 611 - 18 = 593 \text{ psi} < 1,625 = \frac{f'_c}{2}$$

BAR REINFORCING STRESSES

Bar reinforcing steel stresses may be analyzed by rigorous methods; or may be approximated by making several assumptions.

Ignore concrete stress and assume the pile moment is to be resisted by the reinforcing steel. For symmetrical reinforcing it can be assumed that the reinforcing takes compression as well as tension. A simplified equation may be used to determine the tensile reinforcingsteel stress.

Tensile stress $f_s = \frac{Md_s}{I_{bars}\,2} + \frac{V'}{\Sigma A_s}$

Where: d_3 = Distance between center of gravity of bars either side of the pile neutral axis.

$I_{bars} = \Sigma[I_o + A_s(d_s/2)^2] \approx \Sigma A_s(d_s/2)^2$

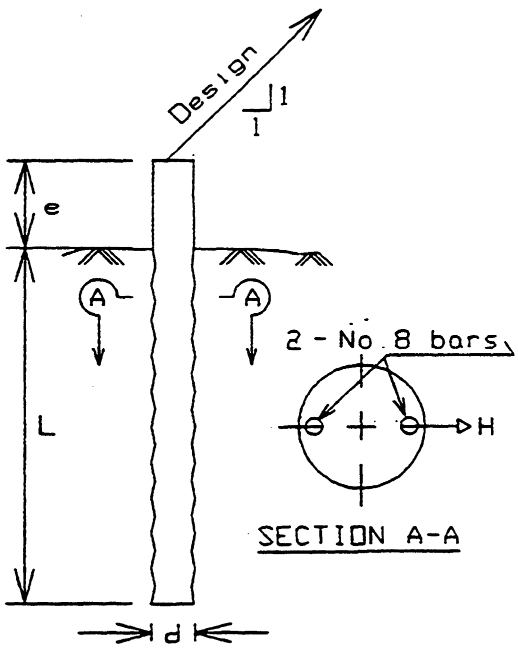
V' = Tensile force (vertical force component) less weight of pile above plane of zero shear, which is located a distance of M_{ULT}/H_{ULT} below the pile top.
Area of bars either side of the neutral axis.

For 2 reinforcing bars, one either side of the pile center line symmetrically placed, the simplified equations is:

$$f_s = \frac{M\frac{d_s}{2}}{(2)A_s\left[\frac{d_s}{2}\right]^2} + \frac{V'}{\Sigma A_s} = \frac{M}{A_s d_s} + \frac{V'}{\Sigma A_s}$$

The allowable stress in the reinforcing steel (F_s) should not exceed 0.70 F_y.

EXAMPLE PROBLEM, BAR REINFORCING STRESS



V_{MAX} = 6,188 Lbs
H_{MAX} = 6,188 Lbs
M_{MAX} = 29,133 Ft-Lbs
L = 8'-0"
pile d = 1'-6"
e = 2'-0"

Reinforcing = 2 - #8 grade 60 bars full length installed 2" clear placed symmetrically along the pile axis.

Single use loading

Determine the bar reinforcing stress in this pile:

Solution:

$d_s = d_{pile} - 2(2\text{" clear}) - 2(d_{bar}/2) = 18 - 2(2) - 2(1.0/2) = 13\text{"}$

$A_s = 0.79 \text{ in}^2$

$\Sigma A_s = 2(0.79) = 1.58 \text{ in}^2$

$V' = 6{,}188 - 1{,}717 = 4{,}471$ Lbs

$$f_s = \frac{M}{A_s d_s} + \frac{V'}{\Sigma A_s} = \frac{29{,}133(12)}{(0.79)(13)} + \frac{4{,}471}{1.58}$$

$$= 34{,}041 + 2{,}830$$

$$= 36{,}871 < 0.7(60{,}000) = 42{,}000 \text{ psi}$$

RESISTANCE TO COMBINED UPLIFT AND HORIZONTAL LOAD

Pile load tests have confirmed that the uplift resistance of piling is increased when the pile is also subjected to a lateral loading. Therefore it is believed acceptable to simply limit combined loadings so as not to exceed the permissible (safety factors considered) H_{ULT} and V_{ULT} loadings.

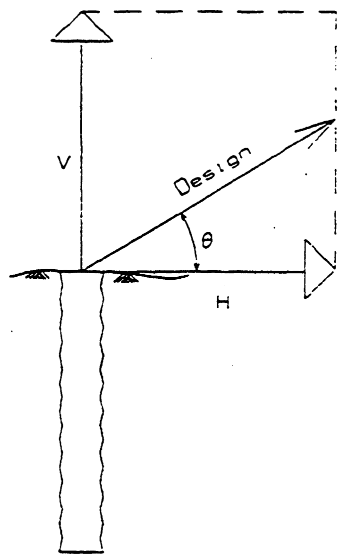


FIGURE 8

Design load is limited to the smaller of either $V/\sin\theta$ or $H/\cos\theta$.

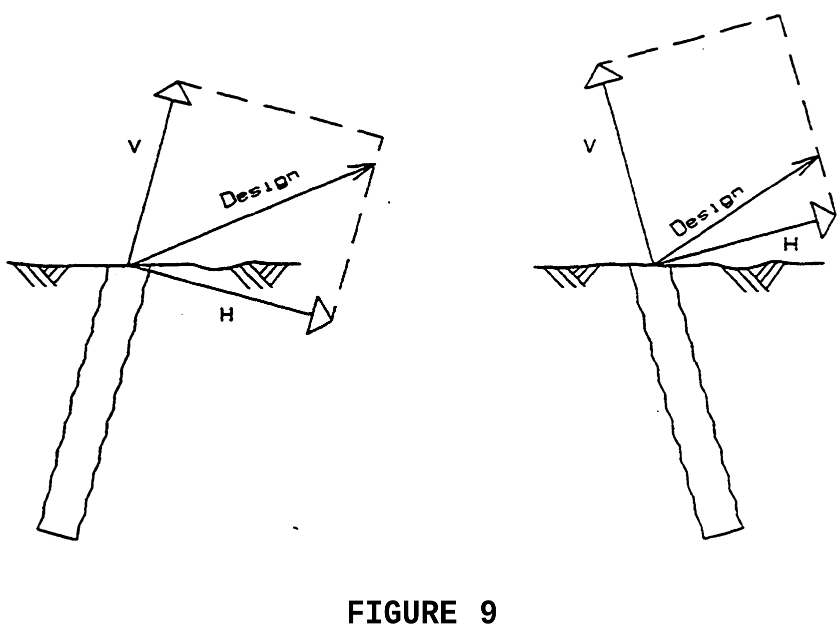


FIGURE 9

Tests have also demonstrated that when the top of the pile is battered toward the load its lateral capacity is substantially greater than when battered away from the load. H and V force components for battered piles are derived in the same manner as for plumb piles. The design load is then limited to the lesser of the H or V load resolved to the slope at which the design load will be acting. Piles battered toward and away from the design loading are depicted in Figure 9.

EXAMPLE PROBLEM. COMBINED UPLIFT AND LATERAL LOAD

A plumb pile has these load capacities:

V_{ULT} = 15,800 Lbs $\qquad$ H_{ULT} = 11,900 Lbs

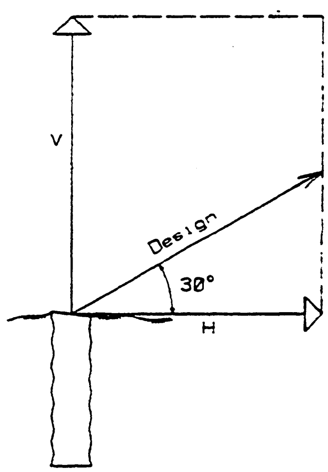
What single use Design load would the pile resist:

a) For a plumb pile?

b) For a pile that is battered 15° towards the load?

Solution:

a) Plumb pile



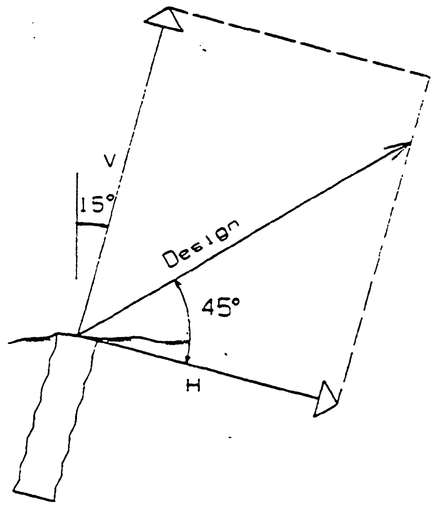
$$\text{Design} = \frac{15{,}800 \text{ Lbs}}{\sin 30^\circ} = 31{,}600 \text{ Lbs}$$

$$\text{Design} = \frac{11{,}900 \text{ Lbs}}{\cos 30^\circ} = 13{,}741 \text{ Lbs}$$

The design loading of 13,741 pounds governs.

$$\textbf{Design working load} = \frac{13{,}741}{2} = 6{,}871 \text{ Lbs}$$

b) Battered pile



$$\text{Design} = \frac{15{,}800}{\text{Sin } 45°}$$

$$= 22{,}345 \text{ Lbs}$$

$$\text{Design} = \frac{11{,}900}{\text{Cos } 45°}$$

$$= 16{,}829 \text{ Lbs}$$

Working Load:

$$\frac{16{,}829}{2} = 8{,}415 \text{ Lbs}$$

The forgoing equations may be used when the horizontal force H is to be less than the computed ultimate lateral force H_{ULT}.

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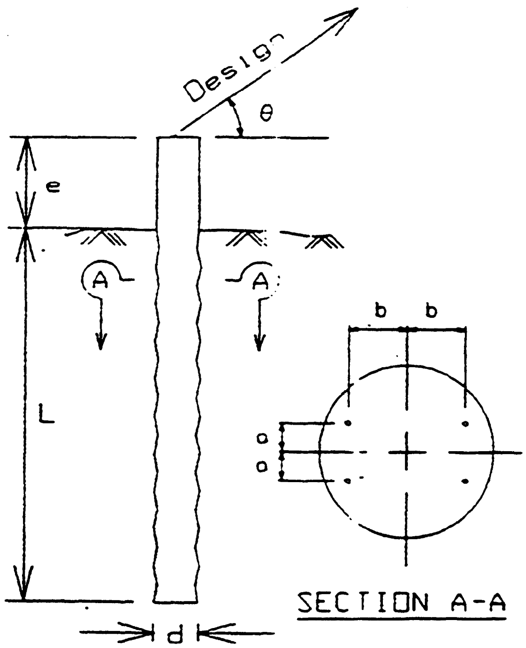
EXAMPLE PROBLEM

A contractor proposes to use an 18 inch diameter poured in-place concrete pile as an anchorage for his falsework cable bracing. Prior. to being used for bracing the falsework this pile will be used as an anchorage for the column reinforcing cage and form.

This anchor pile will be subjected to three short term loads in the same direction.



Sandy Soil (cohesionless)
ϕ = 35°
γ_s = 110 pcf
γ_c = 145 pcf
Minimum f'_c = 3250 psi
2-#5 bars each side of centerline, grade 60, full length
a = 2 in
b = 6 1/4 in
Design = 8,000 Lbs
θ = 35°
e = 1.2 Ft
L = 12 Ft - with lower 2 Ft submerged

Constant Parameters

$K_p = \tan^2(45° + \phi/2) = 3.69$

$\frac{e}{d} = \frac{1.2}{1.5} = 0.8$

$\frac{L}{d} = \frac{12}{1.5} = 8.0$

8.0 < 20 (meets short pile criteria)

$\text{Pile Area} = \frac{\pi d^2}{4} = 254.5 \text{ in}^2$

$\text{Pile } I_g = \frac{\pi d^4}{64} = 5{,}153.0 \text{ in}^4$

$\frac{f'_c}{2} = \frac{3{,}250}{2} = 1{,}625 \text{ psi}$

$V_u = 2\sqrt{f'_c} = 2\sqrt{3250} = 114$ psi

$A_s = 0.31\ in^2 \quad d_{bar} = 0.625$ in

allowable $f_s = 0.70\ f_y = 0.70(60{,}000) = 42{,}000$ psi

1. Clearance

Check distance from center of pile to center of bar.

$$9 \text{ in} - \left\{ \left[(6.25)^2 + (2)^2 \right]^{1/2} + \frac{0.625}{2} \right\} = 2.06 > 2 \text{ in O.K.}$$

$d_s = 2b = (2)(6.25) = 12.50$ in

2. Load Components

H_{DESIGN} = Design cos 35° = (8,000)(cos 35°) = 6,553 Lbs
V_{DESIGN} = Design sin 35° = (8,000)(sin 35°) = 4,589 Lbs

3. Safety Factor

SF = 2.0 + (x-1)(0.25) = 2.0 + (3-1)(0.25)
= 2.5 for lateral soil loading

4. Uplift Capacity Dependent Upon Soil Properties

$S = \beta\sigma_Z$ where:

$Z = 12$ ft with $Z_{dry} = 10'$ and $Z_{wet} = 2'$

$\sigma_Z = (10)(110) + 2(110 - 62.4) = 1{,}195$ pcf

$\beta = 1.5 - 0.315(12)^{1/2} = 0.41 > 0.25$ O.K.

S = .41(1,195) = 490 psf < 4,000 psf O.K.

Net pile shearing resistance = πdzS
$(\pi)(1.5)(12)(490)$
= 27,709 Lbs

Pile weight = $\dfrac{\pi(1.5)^2(13.2)(145)}{4} = 3{,}382$ Lbs

Ultimate load capacity = 27,709 + 3,382 = 31,091 Lbs

Working load = $\dfrac{31{,}091}{2}$ = 15,546 Lbs > 4,589 Lbs O.K.

This soil is capable of resisting the applied vertical load.

5. Lateral Capacity Dependent Upon Soil Properties

$4d = (4)(1.5) = 6.0 < 12'$ (Meets minimum embedment length requirements)

Use weighted average of effective soil densities to account for variable soil layers.

$$\gamma_s = \frac{(10)(110) + (2)(110 - 62.4)}{12} = 99.6 \text{ pcf}$$

From Figure 3: $\dfrac{H_{ULT}}{K_p \gamma_s d^3} \approx 16$

$$H_{ULT} = K_p \gamma_s d^3 (16) = (3.69)(99.6)(1.5)^3(16) = 19{,}846 \text{ Lbs}$$

$$f_g = \left[\frac{H_{ULT}}{1.5\gamma_s d K_p}\right]^{1/2}$$

$$= \left[\frac{19{,}846}{1.5(99.6)(1.5)(3.69)}\right]^{1/2} = 4.90 \text{ feet}$$

$$M_{ULT} = H_{ULT}\left[e + \frac{2f_g}{3}\right]$$

$$= (19{,}846)\left[1.2 + \frac{(2)(4.90)}{3}\right] = 88{,}645 \text{ Ft-Lbs}$$

Working Load Value for $H_{ULT} = \dfrac{19{,}846}{2.5} = 7{,}938 \text{ Lbs} > 6{,}553 \text{ Lbs}$

Working Load Value for $M_{ULT} = \dfrac{88{,}645}{2.5} = 35{,}458 \text{ Ft-Lbs}$

The soil is capable of resisting the applied horizontal load.

6. Pile Adequacy

Pile capacity is to be based on design loads. The lateral force H_{DESIGN} may be substituted for H_{ULT} and M_{DESIGN} for M_{ULT} in the critical soil equations.

$$V_{DESIGN} = 4,589 \text{ Lbs}$$

$$H_{DESIGN} = 6,553 \text{ Lbs}$$

$$f_g = \left[\frac{H_{ULT}}{1.5\gamma_s d k_p}\right]^{1/2}$$

$$f_g = \left[\frac{6,553}{(1.5)(99.6)(1.5)(3.69)}\right]^{1/2} = 2.82 \text{ Ft}$$

$$M_{ULT} = H_{ULT}\left[e + \frac{2f_g}{3}\right]$$

$$M_{DESIGN} = (6,553)\left[1.2 + \frac{(2)(2.82)}{3}\right] = 20,183 \text{ Ft-Lbs}$$

Depth to plane of zero shear of pile $\approx \dfrac{M_{DESIGN}}{H_{DESIGN}} \approx \dfrac{20,183}{6,553} \approx 3.08$ Ft

7. Concrete Stress

$$\text{Pile weight} = (3.08 + 1.2)\gamma_c \pi \left(\frac{d}{2}\right)^2$$

$$= (4.28)(140)(\pi)\left(\frac{1.5}{2}\right)^2 = 1,059 \text{ Lbs}$$

$$V' = 4,589 - 1,059 = 3,530 \text{ Lbs}$$

$$f_c = \frac{Md}{2I_g} - \frac{V'}{A_g}$$

$$f_c = \frac{(20,183)(12)(1.5)(12)}{(2)(5,153)} - \frac{3,530}{254.5}$$

$$= 423 - 14 = 409 \text{ psi} < 1,625 \text{ psi} = \frac{f'_c}{2} \quad \text{O.K.}$$

$$v_u = \frac{V}{0.5bd} = \frac{6,553}{(0.5)(254.5)} = 51 \text{ psi} < 114 \text{ psi}$$

8. Bar Reinforcing Stress

$$f_s = \frac{M}{A_s d_s} + \frac{V'}{\Sigma A_s} = \frac{(20,183)(12)}{2(0.31)(12.50)} + \frac{3,530}{1.24}$$

$$= 31,251 + 2,847 = 34,098 < 0.7(60,000) \text{ psi} \quad \text{O.K.}$$

This pile is capable of resisting the applied loads. The pile is satisfactory for use as designed by the contractor.